HYDRAULIC CAPACITY EVALUATION FOR IMPERIAL BEACH RESORT

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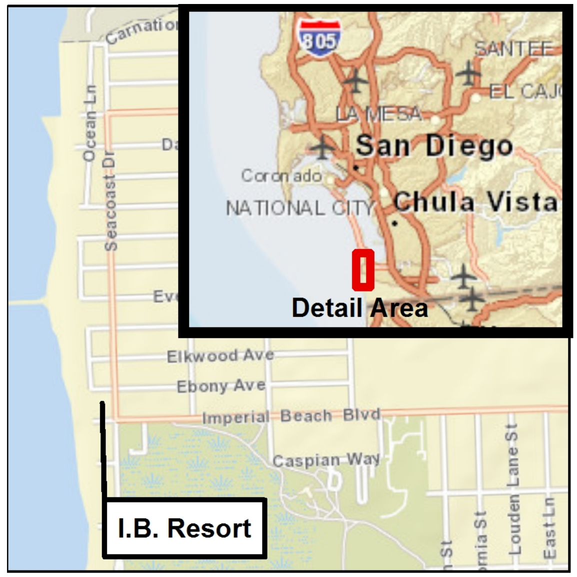
PROJECT DESCRIPTION



A proposed development project located at 1060 Seacoast Drive in the City of Imperial Beach, California, includes the demolition of several existing commercial buildings and the construction of a hotel and resort. The proposed Imperial Beach Resort (IB Resort) includes a bar and kitchen in addition to a 100 room capacity. This report documents the impacts of the anticipated additional flow to the City of Imperial Beach sewer system.

The impacts of the additional flows were calculated by developing a scenario within the existing City of Imperial Beach sewer system hydraulic model, which is maintained by Michael Baker International. This model was developed using the Innovyze InfoSewer software and calibrated with flow monitoring data collected by the city in 2006 to provide an accurate dynamic system model. The model is analyzed under Peak Wet Weather Flows, which are peaked from the Average Dry Weather Flow using an empirical formula. The analysis is performed utilizing the gravity main “range” report feature in InfoSewer. This report assess each pipe over the course of the extended period simulation to identify the worst case scenario down to 1 minute increments. These results take into account backwater affects caused by capacity constraints downstream.

For this project effluent production rates were provided by the Architect for the IB Resort kitchen and bar, calculated based on fixture units; the kitchen and bar will be in operation 18 hours per day. Flows for the hotel rooms were calculated following the Los Angeles Bureau of Engineering Manual Part F200 Sewer Design Guide 1992 (most recent version). This guide provides a detailed list of projected sewer flows generated by public and commercial facilities. The estimated flow rates for the proposed development are as follows:

Table 1: Estimated Flows for the IB Resort

Facility	Unit Flow	Quantity	Additional Flow (gallons / day)
Hotel Rooms	150 gpd/room	100 Rooms	15,000
Kitchen/Bar	27 gpm	18 hours	29,160
Total			**44,160**

The calculated flows were then distributed throughout the day based on a previously determined commercial and residential diurnal curve that has been calibrated for the area. The diurnal curves were developed from flow monitoring data collected in 2006. The distributed flows were then added to

Manhole 1, which is directly adjacent to the proposed project (see Figure 2), and the model was run to simulate a 48 hour period.

The calculated flows were then distributed throughout the day based on a previously determined commercial and residential diurnal curve that has been calibrated for the area. The diurnal curves were developed from flow monitoring data collected in 2006. The distributed flows were then added to Manhole 1, which is directly adjacent to the proposed project (see Figure 2), and the model was run to simulate a 48 hour period. Figure 1 shows the system loading curve over the simulation period; peak loading occurs at approximately the 30th hour of the simulation, approximately between 6:00 and 7:00 am. Therefore this time period is anticipated to be the worst case scenario.

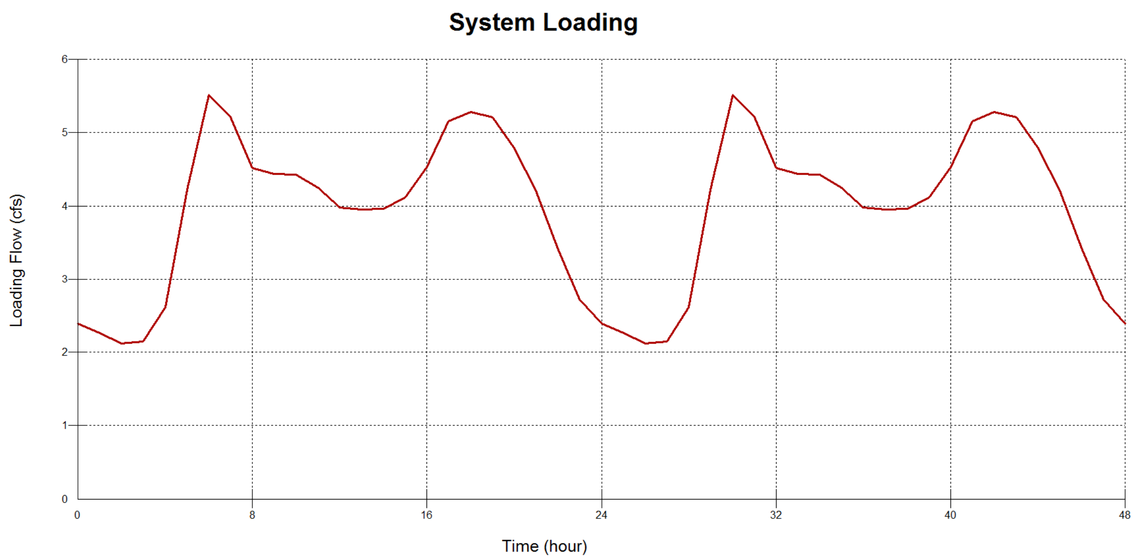


Figure 1: System Loading Curve

CALCULATIONS AND CRITERIA

The flows analyzed were peak wet weather flows for all scenarios. For the following analysis, the gravity main "range" report feature in InfoSewer was utilized. This report assess each pipe over the course of an extended period simulation to identify the worst case scenario down to 1 minute increments. These results take into account backwater affects caused by capacity constraints downstream.

The results are then evaluated based on maximum adjusted velocity (which is the velocity adjusted for backwater affects) and maximum normal depth to diameter ratio (d/D). Per section 1.3.3.3 of the City of San Diego Sewer Design Guide (2013), sewer grades shall be designed for velocities of 3 to 5 feet per second (fps), with minimum velocity of 2 fps and maximum velocity of 10 fps. Per section 1.3.3.3 of the City of San Diego Sewer Design Guide (2013), sewer mains 15 inches and smaller ("small" mains) in diameter shall be sized to carry the projected peak flow at no more than half the diameter of the pipe, or d/D ≤ 0.5. Mains 18 inches and larger ("Large" mains) shall be sized to carry the projected peak flow at no more than three quarters of the pipe diameter, or d/D ≤ 0.75. This information is summarized in Table 2.

Table 2: Normal Depth over Diameter Analysis Criteria

Sewer Main Diameter	Maximum Allowable d/D
< 18"	0.5
≥ 18"	0.75

HYDRAULIC MODEL RESULTS

The result of adding an additional 44,160 gallons per day to the sewer system is shown geographically in Figure 2 and detailed in Table 3. Results were analyzed for both existing and proposed conditions to determine the maximum, or worst case, scenarios for flow (cubic feet per second) and depth-to-diameter ratio. During both existing and proposed scenarios of the extended period flow simulation zero distribution mains greater than 18-inches and five mains less than 18-inches in diameter exceeded the maximum design depth to diameter ratio (d/D). This includes a short 10-inch main that runs between Pump Station B1 and the associated wet well that reaches maximum capacity (d/D = 1) during the simulation period. Based on the configuration of the wet well and lift station it is likely that Pipe 905 is intended to be used as storage.

Table 3: Flow Simulation Model Results

Pipe No.	Dia. (in)	Length (ft)	Slope	US MH	DS MH	Flow (cfs)		Maximum d/D		Design Criteria (d/D)
						Existing	Projected	Existing	Projected	
905	10	5.3	0.534	PS B-1	WW-B1	4.676	4.758	**1.000**	**1.000**	0.50
876	10	62.0	0.018	MH A-1	PS B-1	0.391	0.474	0.304	**0.803**	0.50
504	15	265.0	0.003	MH-409	MH-410	3.154	3.155	**0.736**	**0.736**	0.50
505	15	535.0	0.003	MH-410	MH-190	3.087	3.087	**0.722**	**0.722**	0.50
496	15	321.0	0.005	MH-400	MH-409	3.257	3.258	**0.652**	**0.653**	0.50
240	15	380.0	0.018	DE-57	MH-400	3.271	3.263	0.491	0.492	0.50
816	15	18.0	0.013	MH-188	DE-57	3.292	3.293	0.473	0.473	0.50
242	15	300.0	0.018	MH-190	MH-193	3.088	3.087	0.410	0.410	0.50
426	21	299.0	0.002	MH-326	MH-327	3.744	3.751	0.558	0.559	0.75
428	21	293.0	0.002	MH-327	MH-329	3.824	3.831	0.537	0.538	0.75
414	21	140.0	0.001	MH-317	MH-318	3.109	3.108	0.519	0.519	0.75
798	10	128.0	0.005	MH-1	MH A-1	0.319	0.401	0.305	0.344	0.50
415	21	300.0	0.002	MH-318	MH-326	3.116	3.116	0.499	0.493	0.75
245	21	132.0	0.002	MH-195	MH-317	3.099	3.099	0.475	0.474	0.75
803	18	148.0	0.014	MH-193	MH-195	3.089	3.088	0.425	0.424	0.75
780	21	62.5	0.023	MH-329	PS-8	3.824	3.831	0.269	0.269	0.75

As seen in Table 1, the changes in maximum flow and d/D ratio are limited for all pipes except number 876. Pipe 876 is directly upstream of Pipe 905 and sees respective increases of 0.08 cfs and 0.5 in d/D ratio under maximum conditions.

The pump stations were evaluated and the results are shown on the next page in Table 2. The capacity for Pump station 1B is exceeded under peak flow conditions.

CONCLUSIONS

The simulation of the City's existing sewer system found limited capacity, as evidenced by five pipes in violation of the design depth-to-diameter ratio, including one completely full pipe. Pump station 1-B was also unable to meet the demand. For the proposed Imperial Beach Resort, total system loading increased 1.6% and one additional pipe exceeds the design criteria. However the additional pipe exceeding design capacity is directly upstream from the completely full pipe, therefore this result was anticipated.

The five pipes shown in the table need to be upsized and Pump station 1-B needs to be replaced or upsized to meet the additional flows from the Imperial Beach Resort. The City of Imperial Beach is studying solutions to the sewer system deficiencies but does not currently have plans to increase the pump station capacity.

Table 4: Flow Simulation Pump Stations 1B and 8

Pump Station ID		**PS1B**	**PS8**
Wet Well ID		**WW1B**	**WW8**
Flow Calculations:			
Simulated Peak Flow (Q_p)	ft^3/sec	4.75	5.56
Peaking Factor (k)		2.4	2.4
Peaking Factor (p)		0.89	0.89
Avg Flow (Q_A)	ft^3/sec	2.4	2.86
Design Pump Flow (Q_p)	ft^3/sec	**2.68**	8.39
Pumps Required for Peak Flow:		**2**	**2**
Wet Well Calculations:			
Manhole Diameter	ft	13.35	8.96
Calculated Specific Volume	cu. ft. / ft.	139.9	62.96
No of Duty Pumps	total pumps	1	1
No of Standby Pumps	total pumps	1	1

Max. # of Pump Cycles	per hr per pump	10	10
Min. Req'd Oper. Volume	ft^3	0.54	1.68
Max. Wet Well Storage Time	min	30	30
Max. Wet Well Volume	ft^3	1,259.78	251.99
Operating Volume Selected	ft^3	153.97	125.99
Active Depth	ft	1.1	2
Pump Cycling Frequency:			
Avg Flow			
Avg. Flow Time to Fill	min	1.07	0.73
Avg. Flow Time to Pump Down	min	9.32	0.38
Avg. Flow Total Cycle Time	min	10.39	1.11
1 Pump Rotation	cycles/hr	5.78	**53.86**
2 Pump Rotation	cycles/hr/pump	2.89	**26.93**
Peak Flow			
Peak Flow Time to Fill	min	0.54	0.38
Peak Flow Time to Pump Down	min	**PS Capacity Exceeded**	0.74
Peak Flow Total Cycle Time	min	n/a	1.12
1 Pump Rotation	cycles/hr	n/a	**53.58**
2 Pump Rotation	cycles/hr/pump	n/a	**26.79**

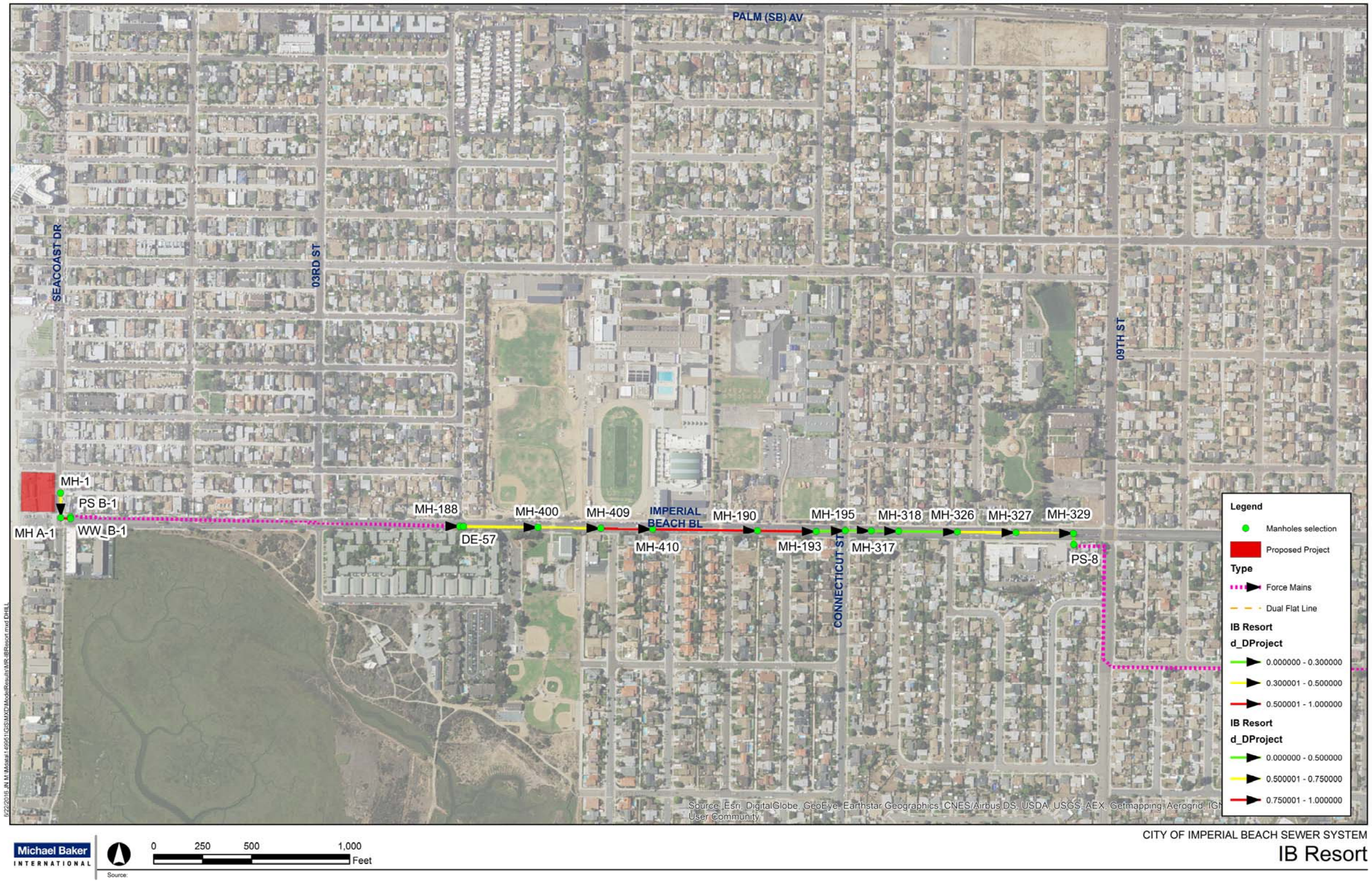


Figure 2. Flow Path Capacity for the Imperial Beach Resort